**من ذكريات العمل**

**مشروع قناطر نجع حمادى الجديدة**

**مشكلة انهيار ميول قناة التحويل**

**(Study of Slope Failures During Excavation of the Diversion Canal)**

**مقدمه:**

تعتبر قناة التحويل من اهم الاعمال المؤقتة فى المشروع ( وهى ثانى تحويلة لمجرى النيل فى التاريخ بعد التحويل الاول اثناء انشاء السد العالى ) فهى التى ستقوم مكان نهر النيل فى الوفاء بالتزاماته فى نقل المياه لجميع اغراض الزراعة والشرب والملاحة وهي مصممه على استيعاب أعلى التصرفات المطلوبة وكذلك تحمل جميع انواع العائمات من بواخر سياحيه وصنادل والمفترض انها ستقوم بكل هذه المهام حوالي اربعة اعوام من عمر المشروع وهي كإجراء مؤقت خلال فتره انشاء القناطر الجديدة، كان لا بد من تحويل النهر. وطبقا للتصميمات فقد تم اختيار البر الغربي للمشروع بمنطقة النجمة و الحمران بقنا مكان لقناة التحويل.

بعض البيانات عن قناة التحويل

- الطول المتوسط 1.1 كم
- عرض القاع 125 مترا
- مساحة التحويلة 25 هكتار ( 60 فدان)
- تتحمل تصرف 2900 م3/ث
- منسوب القاع (52.00)
- الميول الجانبية التصميمية 3H:1V
- منسوب الجسور 65.00 & 67.50 للبرين الأيمن والايسر على التوالي
- بلغ أجمالي قيمة اعمال قناة التحويل 34.00 مليون جنيه مصرى و 2.25 مليون يورو بأسعار 2002.

تشمل الاعمال :

- أعمال كشط وإزالة الجذور الموجودة بمنطقة العمل لمساحة حوالى 25.00 هكتاروتم تشوينها فى الأماكن المخصصة لها وهى تتمثل فى جذور القصب والنباتات الموجودة بالمنطقة والحشائش وخلافه وذلك فى حدود 10 سم تقريبا وذلك تمهيداً لوضعها فى البرك والأماكن المنخفضة بمنطقة الاستصلاح.
- أعمال حفر 40 سم تربة زراعية بلغت كميتها حوالى 100.000 متر مكعب ولها مكان مخصص للتشوينات وهذه النوعية من التربة تم تحديدها بمعرفة خبراء فى الزراعة وهو العمق المناسب لنمو

النباتات (الطبقة الخصبه) وتم تغطيتها بالمشمع طوال فترة العمل حتى يتسنى اعادتها الى أماكنها فى أرض المزارعين المنزوعة نزع مؤقت والتى سيعاد تأهيلها (ضمن إجراءات حماية البيئة)

- أعمال الحفر الجاف باستخدام الحفارات وبلغت الكمية حوالى 1.400.000 (مليون وأربعمائة ألف) متر مكعب ياستخدام الحفارات العملاقة وتم تقسيم التحويلة الى ثلاثة أقسام رئيسية هى الجزء الشمالى الجزء الجنوبى الجزء الأوسط. بدأ الحفر أوائل يوليو فى الجزء الشمالى الملاصق للنيل لامكانية فتح ممر لكراكة البحرية التى ستبدأ من الشمال.
- أعمال التكريك و الحفر تحت الماء باستخدام الشفاطات Dredgers و بلغت كمية التكريك 1,565,000 (مليون و خمسمائة و خمسة وستون الف متر مكعب)
- أعمال الحمايات للقاع والجوانب من احجار ال RIP RAP وشرائح ال Geotextile.
- واعتبر المنسوب (60.40) هو الفاصل بين الحفر الجاف والتكريك
- ويجب أن تنتهي هذه الأعمال في التاريخ الحاكم في 30-10-2003 نظرًا لارتباطها بأعمال إنشاء السد المؤقت ال (Coffer Dam) وذلك لتنفيذ الأعمال المدنية للأساسات والمنشآت( المفيض و الاهوسة الملاحية) ومحطة الكهرباء.
- بدأت اعمال الحفر الجاف عقب استلام الموقع في يوليو 2002 باستخدام الحفارات.
- ثم بدأ العمل في التكريك باستخدام الشفاطات في أكتوبر 2002
- سارت الاعمال بشكل جيد في اعمال الحفر الجاف حتى النهاية
- اما بالنسبة الى اعمال التكريك فقد حدثت مشاكل اثناء التنفيذ سنتناولها في هذا المنشور .

**انهيار ميول قناة التحويل أثناء التكريك**

يعد تكريك المجارى المائية جزءًا أساسيًا من المشاريع الهيدروليكية، لكنه يواجه تحديات كبيرة، خاصة في المناطق التي تحتوي على تربة غير متماسكة حيث واجه التنفيذ عدة مشاكل تتعلق بانهيار ميول القناة أثناء أعمال التكريك، مما تسبب في تأخير التاريخ الحاكم من 30-10-2003 الى 18-12-2003 أي حوالى 48 يوما مما تسبب في مشاكل فنية وتعاقدية كبيرة نظرا لارتباط هذا الموعد بأنشطة أخرى للاعمال المدنية والهيدروليكيه.

استخدم المقاول في اعمال التكريك عدد 2 شفاط هما الكراكة الصفا و الكراكة المروة

المواصفات الفنيه لهما كالاتى:

**The main technical data for the dredgers were:**

**El Safa:**

**Manufacturer:** **Ellicott-USA**

Ser 3000 Super Dragon

Total 3250 HP

Main P 2250 HP

Cutte 480 HP

Max. 18 m

Suc./Disposal Pipe Diameter: 24'' (600 mm)

Length of Disposal Pipeline: 3000 m

**EL Marwa**

**Manufacturer:** **Ellicott-USA**

**Series:** **1870 Super Dragon**

**Total Prime Movers:** **1755 HP**

**Main Pump:** **1280 HP**

**Cutter:** **250 HP**

**Max. Digging Depth:** **15.5 m**

**Suc./Disposal Pipe Diameter:** **20'' (500 mm)**

**Length of Disposal Pipeline:** **2000 m**

**سرد تاريخي موجز عن حوادث الانهيارات والإجراءات التي تمت ثم نتبعه بالأسباب وطرق العلاج والتوصيات**

- استيقظ الجميع صبيحة يوم الاحد الموافق 10 نوفمبر 2002 على انهيار كبير بالميل يالبر الأيسر فى شمال التحويلة كان شيئا غريباً وغير متوقع الحدوث ومع ذلك استمر المقاول فى العمل حتى فوجئ الجميع بحدوث انهيار اخر كبير فى 22 ديسمبر من نفس العام 2002 .. بدأ القلق يتسرب الى الجميع إن خطر كهذا كفيل بتأخر انتهاء الأعمال بالتحويلة وكذلك سيترتب عليه تأخر جميع الأعمال المترتبة عليه من سدود مؤقتة وأهم من ذلك المنشأ الرئيسى نفسه وهو القناطر الجديدة.

- بدأت أصابع الاتهام تشير الى سائقى الحفارات المائية وطريقة التشغيل التى يتبعها المقاول وبدأ المقاول فى اتخاذ إجراءات احتياطية تتمثل فى اتخاذ بعد كافى عن الميل وتقليل سرعة المص بالقرب من الميول أخذ المقاول يشكو من التربة ونوعيتها وعدم تمكنه من تنفيذ الميول التصميمية 1:3 وتم تبادل الاتهامات بين جميع الاطراف وكل طرف يحمل الأخر المسئولية
- اختلف المكتب الاستشاري المصمم والمشرف على التنفيذ مع المقاول حول أسباب هذه الانزلاقات، حيث يرى المقاول أن السبب هو عدم ملاءمة التصميم لنوعية التربة في الموقع، بينما يرى الاستشاري أن السبب يعود إلى استخدام الجرافتين وطريقة التشغيل التي يعمل بها المقاول.
- فى الفترة من 21 إلى 25 يناير 2003 قام المكتب الإستشارى لاماير بدعوة خبير التكريك العالمى د/ فولكر باتزولد إلى زيارة الموقع لإبداء ملاحظاته على:-

1. طريقة التشغيل المستخدمة فى أعمال التكريك.
2. تحسين آداء أعمال التكريك.
3. تقدير اتزان الميول التصميمية.

- وقدم تقريره فى 4 فبراير 2003 وتضمن التقرير النقاط التالية:-
- المقاول لديه كراكتين هما الصفا والمروة ولقد خصص المقاول الكراكة المروة لتكريك الميول والكراكة الصفا لتكريك الجزء الأوسط وفيما يبدو فإن هاتين الكراكتين قديمتين.
- قوة السحب وحجم القواطع غير مناسبة.
- طريقة تكريك الميول...يستخدم المقاول الكراكة المروة فى عمل الميول ولفت نظر الخبير أن عرض الخطوة 1.5م وأقل عرض لها 0.9م وأن سرعة الدوران حوالى 20م/دقيقة ووصلت فى 2003/1/23 إلى 29م فى الدقيقة وهى سرعة عالية.
- هناك صعوبات في تجريف الميول بالتصميم المطلوب، مما يتطلب تعديلات في طرق العمل.
- نظام التحكم في تشغيل الجرافات ضعيف وقديم وغير كافٍ، مما يؤثر على جودة العمل

التوصيات:

- تحديث المعدات واستخدام تقنيات حديثة لتحسين الأداء
- تعديل طرق التجريف لتناسب ظروف التربة، خاصة في مناطق الميول
- استخدام أنظمة تحكم حديثة لضمان دقة العمل وتسهيل المراقبة على الإنترنت
- زيادة المراقبة من قبل الاستشاريين لضمان الالتزام بالمواصفات والجدول الزمني
- حدثت انهيارات أخرى 11-2-2003 و 16-3-2003 و 17-5-2003
- 11-05-2003 وافق المالك على طلب الاستشاري بتعديل الميول لتصبح 5:1 بدلا من 3:1
- حدث انهيار أخر يوم 10-7-2003 بطول حوالى 80 متر وبعمق 6 امتار في البر الأيمن قريبا من الستارة المؤقتة

- حدث الانهيار الأكبر يوم 21-7-2003 في البر الأيمن أيضا شمال الانهيار السابق ولكن أخطر حيث وصل الى الستارة المؤقتة وأصبحت مكشوفة بطول 30 مترا
- أصبح الشغل الشاغل الان هو المحافظة على سلامة الستارة واللحاق بموعد تشغيل قناة التحويل في 30-10-2003.
- فى اغسطس 2003 اقترح المقاول عمل تجربة لتشكيل الجانب الرئيسي في الجزء الشمالي من قناة التحويلة باستخدام حفارات ذات ذراع طويل (Long Arm) محملة على بارجة الا انها فشلت بسبب صعوبة تثبيت البارجة بدقة، مما أدى إلى حفر زائد وتشققات و قيام الاستشاري بإعطاء تعليمات للمقاول بإيقاف التجربة حيث شعر الجميع إلى أن هدف المقاول من التجربة كان إثبات أن الجرافتين وطريقة التشغيل ليستا السبب في الانزلاقات.
- كما قام المالك أيضا بالاستعانة بالأستاذ الدكتور عبد الفتاح أبو العيد لدراسة المشكلة ومعرفة الأسباب ومدى مناسبة تصميم الاستشاري وكذلك معدات التكريك وأسلوب التشغيل الذى قام به المقاول.

وكان رأى د أبو العيد السبب الرئيسى للانهيارات قد يعود الى قوة الشفط الافراط في القطع مما أدى الى فقدان القوة والتماسك في الجزء الرملي المغمور من الميول كما قد يعود الى التسرب من الأراضي المجاورة نتيجة اعمال الرى .

وبعد عدة اجتماعات مع كل الأطراف تم الاتفاق على الاتى:

1- ابعاد الشفاط الصفا الأكبر سعة عن العمل بقرب الميول
2- متابعة اعمال التكريك بالليل ووضع مهندس من الاستشاري لمتابعة عمل الشفاط
3- تقليل الميل من 3:1 الى 5:1 وفي المخرج الشمالى للقناة يمكن زيادته الى 7:1
4- العمل في هذه المنطقة بالحفارات ذات الزراع الطويل long arm
5- سرعة ردم المناطق المنهارة وحمايتها بالاحجار RIP-RAP

استمر العمل بالتكريك على ضوء ما تم الاتفاق عليه وأراء السادة الخبراء الاستشاريون حتى انتهاء العمل فى 2003/12/18 اى بتأخير 48 يوما مما أدى الى قيام المالك بتوقيع غرامة تأخير وقدرها 490 الف يورو لاقتناع المالك ان ما حدث هو نتيجة سوء التشغيل من قبل المقاول مما أدى اهذه الانهيارات.

وستتعرض في ما يلى الأسباب الرئيسة للانهيارات والتوصيات على ضوء التقارير والمكاتبات في هذا الشأن :

## 1. <u>الأسباب الرئيسية لانهيارات الميول</u>

نجمت الانهيارات عن مجموعة متضافرة من العوامل، أبرزها:

<u>طبيعة التربة</u>: تتكون التربة في منطقة القناة، خاصة في الطبقات العلوية ("طمي النيل")، من رمال ناعمة سائبة وغير متماسكة ذات كثافة منخفضة. هذه التربة عرضة لـ "تسييل التربة (Liquefaction) "وفقدان

الاستقرار بسرعة تحت تأثير الاهتزازات أو التغيرات في الضغط الناتجة عن أعمال التجريف. كما ساهم وجود طبقات طينية رخوة أو رملية رقيقة متداخلة في تفاقم المشكلة.

قصور معدات التجريف: تم استخدام جرافات شفط ذات قاطع (CSDs) مثل "المروة" و "الصفا"، والتي كانت ذات تصميم قديم تفتقر إلى أنظمة التحكم الدقيقة وأنظمة المراقبة الحديثة. هذا جعل من الصعب التحكم في كمية المواد المجرفة بدقة وتجنب القطع الزائد، خاصة عند تجريف المنحدرات في التربة الرملية المفككة.

أسلوب التشغيل: عملت الجرافات في بعض الأحيان بشكل متعامد وقريب جدًا من المنحدرات (مسافة تصل إلى 10 مترًا)، مما زاد الضغط على التربة. كما أن إزالة كميات كبيرة من التربة دفعة واحدة بدلاً من التجريف التدريجي زاد من الإجهادات على الميول.

التصميم الأولي للميول: كانت زاوية الميل الأصلية للمنحدرات تحت الماء مثل 1 رأسي إلى 3 أفقي – (V1:3H)حادة جدًا بالنسبة لطبيعة التربة غير المستقرة، مما زاد من احتمالية الانزلاق.

غياب أنظمة المراقبة الفعالة: افتقار الجرافات والموقع لأنظمة مراقبة جيوتقنية حديثة حال دون الكشف المبكر عن بوادر عدم الاستقرار، مما جعل الاعتماد على الملاحظات الميدانية غير كافٍ لمنع الانهيارات المفاجئة.

**2-الحلول والإجراءات المتخذة :لمعالجة المشكلة وضمان استقرار القناة، تم اتخاذ الإجراءات التالية:**

- الإصلاح الفوري للانهيارات: تم ملء الفجوات الناتجة عن الانهيارات فورًا باستخدام مواد الرمل والحصى ، مع إضافة طبقات حماية من الجيوتكستايل والحجارة (Riprap) عند الحاجة.
- تعديل تصميم الميول: بناءً على الحوادث المستمرة وتوصيات الخبراء، تم تعديل ميل المنحدرات تحت الماء لتصبح أقل حدة. تم تغيير الميل من 3H:1V الى 5H:1V بشكل عام أسفل منسوب 58 مترًا فوق سطح البحر، وفي بعض المناطق المتسعة عند المخرج الشمالى تم تعديله إلى 7:1 لزيادة الاستقرار .
- تعديل مواصفات مواد الحماية: أدى تضييق المقطع العرضي للقناة (بسبب تعديل الميل) إلى زيادة سرعة تدفق المياه (من حوالي 1.78 م/ث إلى 2.05 م/ث). استلزم ذلك استخدام مواد حماية (Riprap)ذات حجم أكبرR6) ، (D50=150mm لتحمل السرعات الأعلى.
- استبدال الجيوتكستايل (Geotextile) في بعض المناطق ذات التيارات القوية بطبقات إضافية من الرمل والحصى أو طبقة R6 بسمك 0.4m .
- تعديل منهجية التجريف: أوصي بتقليل حجم القطع، استخدام جرافات أصغر للميول، ونقل المواد لتزيلها الجرافات الكبيرة، والعمل بزوايا أكثر تدرجًا وبسرعات أبطأ.

على الرغم من التحديات الكبيرة المتعلقة بانهيارات الميول الناتجة عن مزيج من ظروف التربة الصعبة وقيود المعدات وأساليب التشغيل، تمكن فريق المشروع من خلال الاستشارات الفنية، وتعديل التصميمات،

وتنفيذ الإجراءات التصحيحية السريعة، من احتواء المشكلة وإكمال قناة التحويل وان التأخير لمدة 48 يوما يعد نجاح في ظل هذه الظروف.

الحديث عن قناة التحويل في مشروع قناطر نجع حمادى يطول وأنها عمل وان كان مؤقتا الا انه يعتبر من الاعمال الرئيسية وبه من الاعمال من حيث طريقة التنفيذ واعمال الحماية والاعمال الترابية يحتاج الى أكثر من بوست وان ما ذكر في البوست الحالي خاص بمشكلة انهيار الميول.

يهدف هذا البوست الى التذكير بالمشكلة والظروف التي تحيط بأعمال التنفيذ في هذا الوقت وشرح الأسباب وتقديم تحليل مختصر، الإجراءات التي تم اتخاذها لمعالجتها، والتعديلات التصميمية التي تم تنفيذها لضمان استقرار الميول.

شكر موصول للزميل العزيز السيد المهندس /عصام عبد الكريم المدير العام الحالي لقناطر نجع حمادى للمراجعة وأضافه بعض التفاصيل واللمسات الأدبية مع كونه أحد المشاركين لهذا الحدث .

للمزيد من التفاصيل لمن أراد الاستزادة حول هذا العمل مرفق موضحا به:

تقرير الخبير من الكتب الاستشاري لاماير

تقرير د عبد الفتاح أبو العيد

بعض الرسومات والصور التي توضح الانهيارات.




2002-11-10


2003 5 12


2003 5 12


2003 7 15


2003 7 21



2003 7 21


11 5 2003


3 3 2003

# قطاع عرضى يبين مكونات التحويلة.

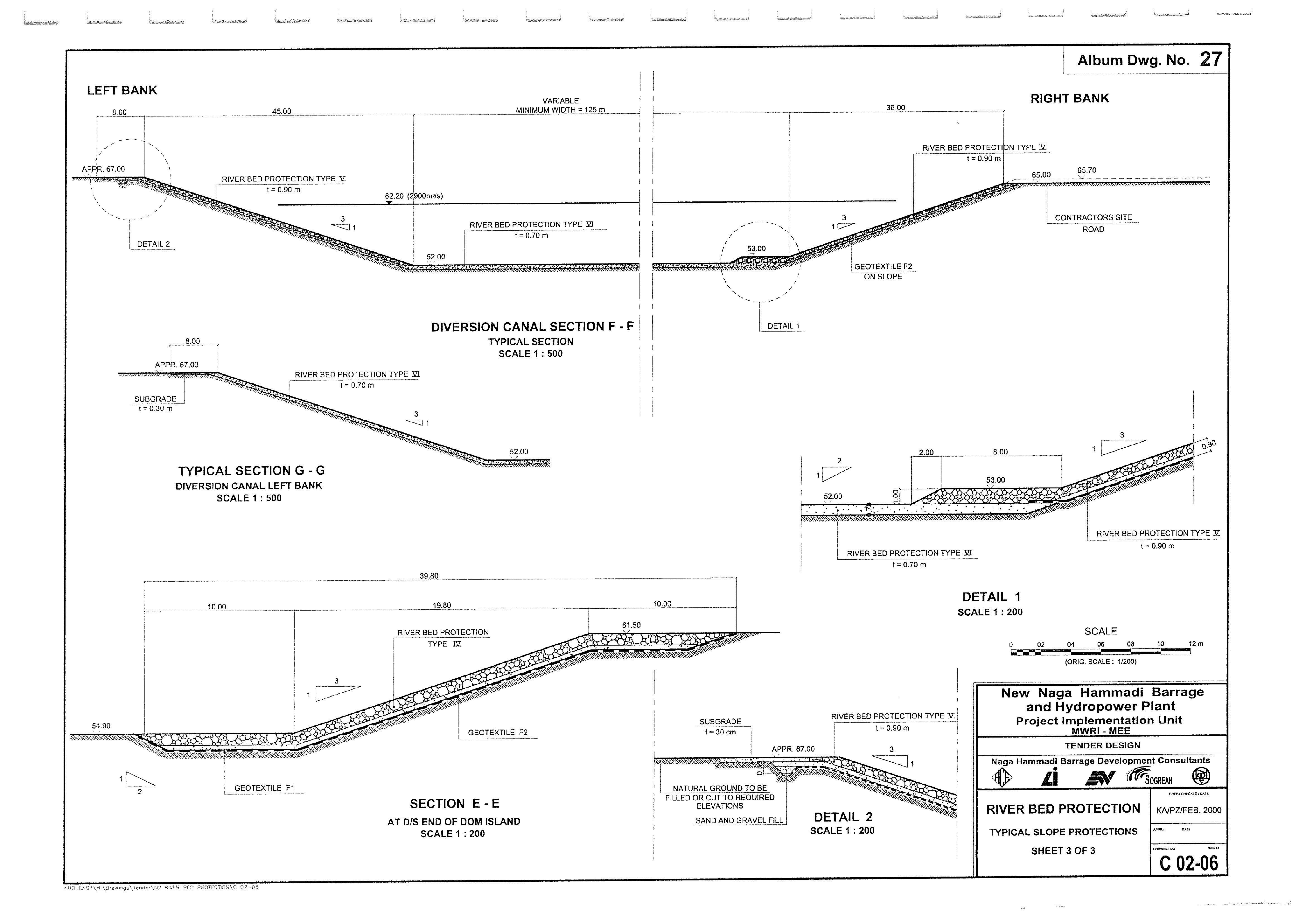
Put Rocks

Geotextil

Slop Protection

Bed Protection

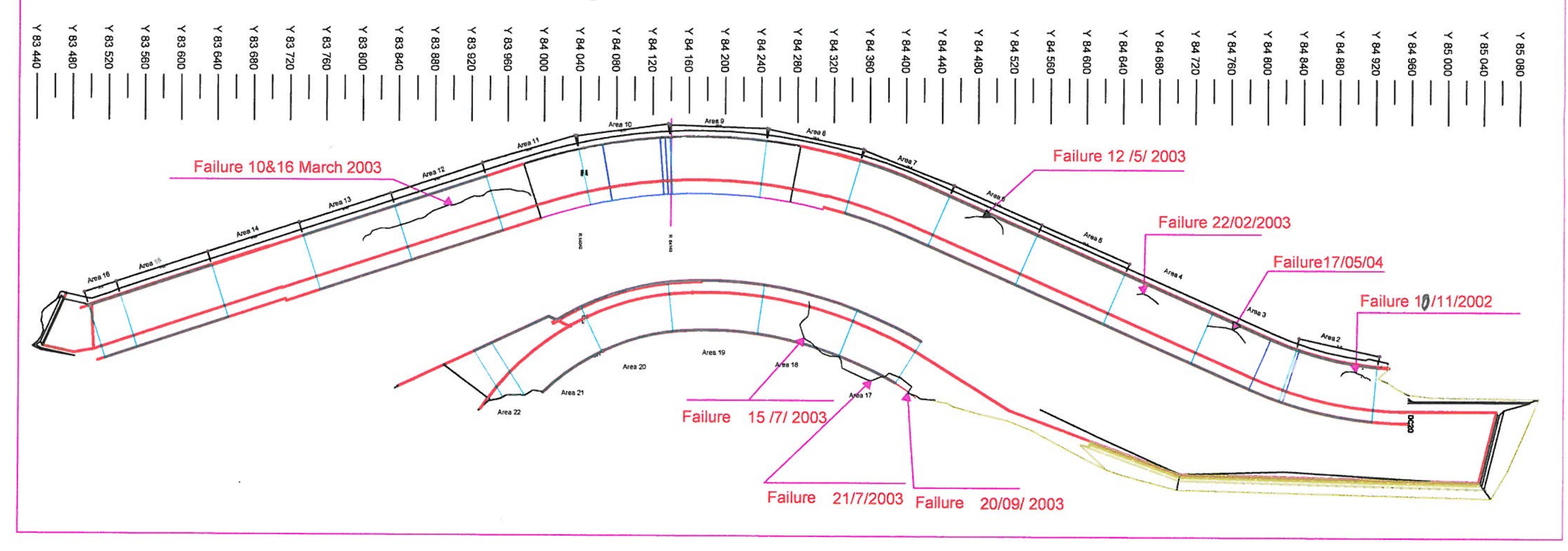
Excavation

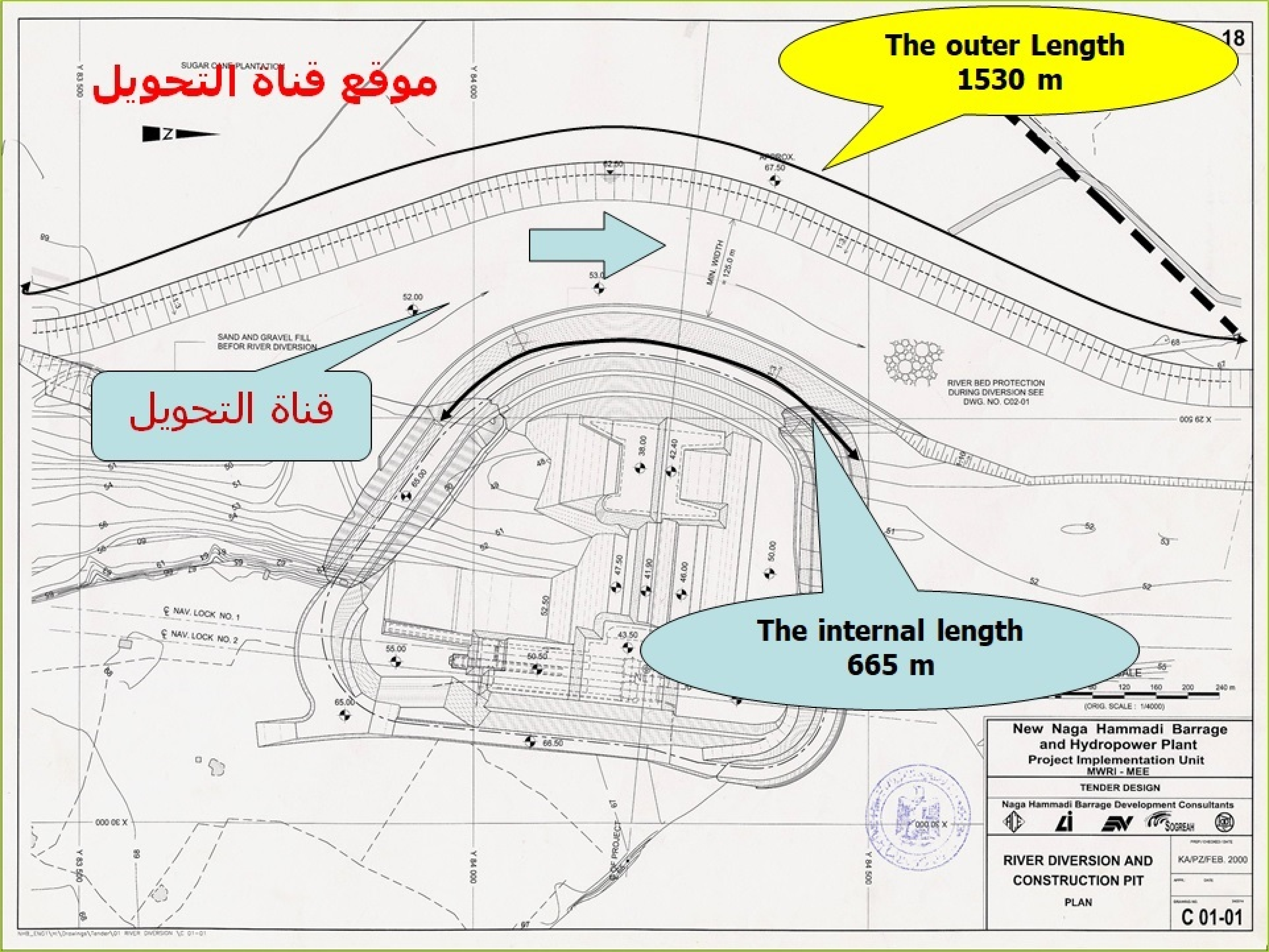


Album Dwg. No. 27
LEFT BANK
RIGHT BANK
8.00
45.00
VARIABLE
MINIMUM WIDTH = 125 m
36.00
APPR. 67.00
RIVER BED PROTECTION TYPE Ⅴ
t = 0.90 m
62.20 (2900m³/s)
RIVER BED PROTECTION TYPE Ⅵ
t = 0.70 m
52.00
53.00
65.00
65.70
CONTRACTORS SITE
ROAD
GEOTEXTILE F2
ON SLOPE
DETAIL 2
DETAIL 1
DIVERSION CANAL SECTION F - F
TYPICAL SECTION
SCALE 1 : 500
SUBGRADE
t = 0.30 m
TYPICAL SECTION G - G
DIVERSION CANAL LEFT BANK
SCALE 1 : 500
2.00
1.00
0.90
DETAIL 1
SCALE 1 : 200
39.80
10.00
19.80
61.50
RIVER BED PROTECTION
TYPE Ⅳ
54.90
GEOTEXTILE F2
GEOTEXTILE F1
SECTION E - E
AT D/S END OF DOM ISLAND
SCALE 1 : 200
SUBGRADE
t = 30 cm
NATURAL GROUND TO BE
FILLED OR CUT TO REQUIRED
ELEVATIONS
SAND AND GRAVEL FILL
DETAIL 2
SCALE 1 : 200
SCALE
0 02 04 06 08 10 12 m
(ORIG. SCALE : 1/200)
New Naga Hammadi Barrage
and Hydropower Plant
Project Implementation Unit
MWRI - MEE
TENDER DESIGN
Naga Hammadi Barrage Development Consultants
SOGREAH
RIVER BED PROTECTION
TYPICAL SLOPE PROTECTIONS
SHEET 3 OF 3
KA/PZ/FEB. 2000
C 02-06
NHB_ENG1\H:\Drawings\Tender\02 RIVER BED PROTECTION\C 02-06

# DIVERSION CANAL

## Main Slope Failure Locations



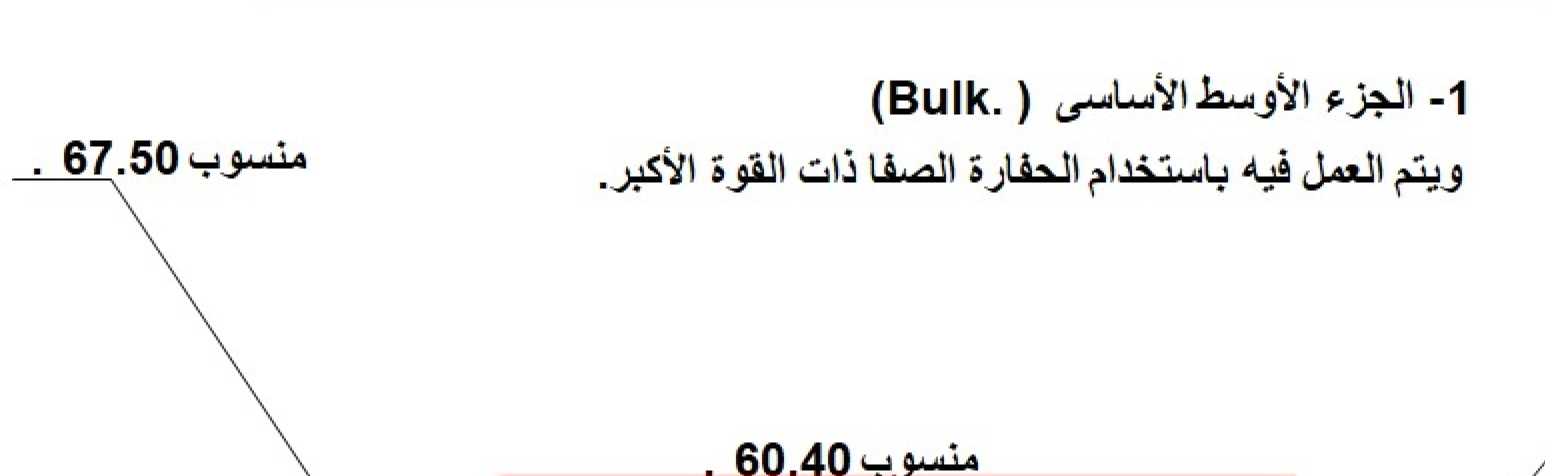


18
موقع قناة التحويل
The outer Length
1530 m
قناة التحويل
The internal length
665 m
SUGAR CANE PLANTATION
SAND AND GRAVEL FILL
BEFOR RIVER DIVERSION
MIN. WIDTH = 125.0 m
APPROX. 67.50
RIVER BED PROTECTION
DURING DIVERSION SEE
DWG. NO. C02-01
NAV. LOCK NO. 1
NAV. LOCK NO. 2
(ORIG. SCALE : 1/4000)
New Naga Hammadi Barrage
and Hydropower Plant
Project Implementation Unit
MWRI - MEE
TENDER DESIGN
Naga Hammadi Barrage Development Consultants
SOGREAH
RIVER DIVERSION AND
CONSTRUCTION PIT
PLAN
KA/PZ/FEB. 2000
C 01-01

## تم تقسيم القطاع المائى تحت الماء الى قسمين أساسيين.

1- الجزء الأوسط الأساسى (Bulk.)

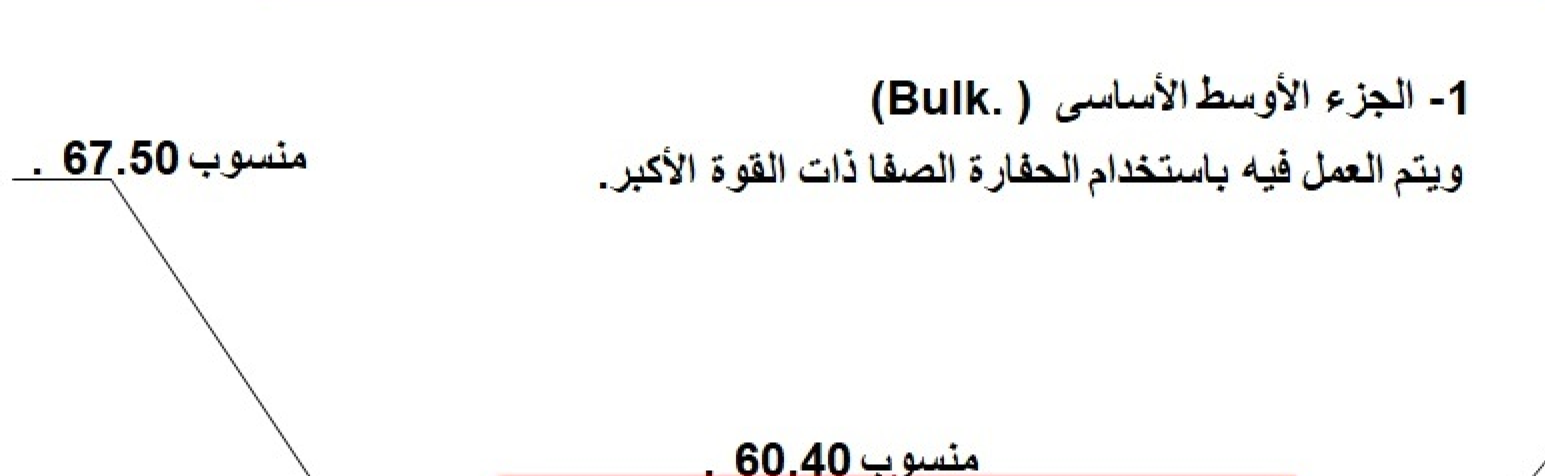
ويتم العمل فيه باستخدام الحفارة الصفا ذات القوة الأكبر.



## تم تقسيم القطاع المائى تحت الماء الى قسمين أساسيين.

**1- الجزء الأوسط الأساسى ( .Bulk)**

ويتم العمل فيه باستخدام الحفارة الصفا ذات القوة الأكبر.

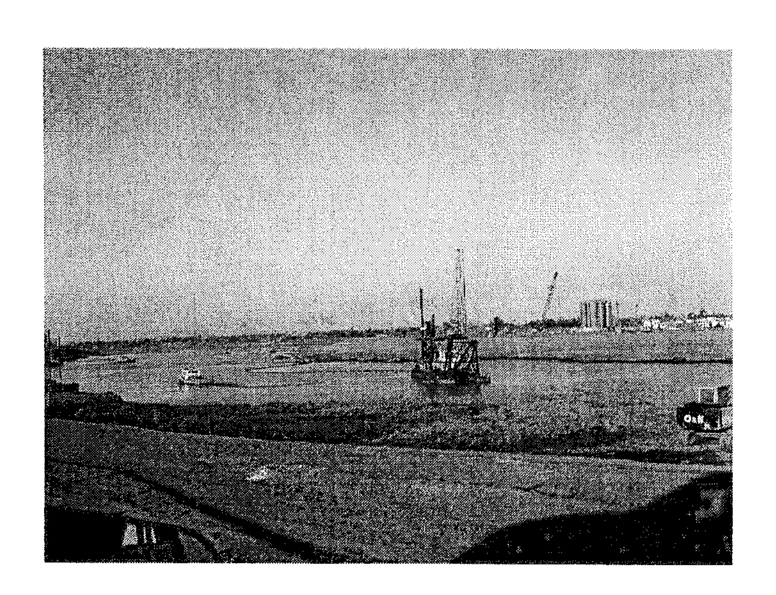


# REPORT

**Site visit of dredging works**
**for**
**New Naga Hammadi Barrage and Hydropower Project**

**on behalf of**

**NAGA HAMMADI BARRAGE DEVELOPMENT CONSULTANTS (NHBDC)**
**Naga Hammadi/Egypt**




by
Dr.-Ing. Volker Patzold
Dredging expert
c/o



**INGENIEURBÜRO DR.- ING. V. PATZOLD**

**Naßbaggerei, Tagebau auf Steine & Erden, Umwelttechnik**

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**January 2003**

**VP Proj Nr. 03-917-4005-4082**

LIST of CONTENTS

| | | |
|---|---|---|
| 0 | **SUMMARY** | 3 |
| 1 | **INTRODUCTION** | 4 |
| 2 | **WORKING METHOD** | 4 |
| 2.1 | **Dredging equipment** | 4 |
| 2.2 | **Slope dredging** | 6 |
| 2.3 | **Discharge pipe and Reclamation area** | 8 |
| 2.4 | **Positioning and work control** | 9 |
| 2.5 | **Construction Schedule** | 10 |
| 3 | **PROPOSALS ON WORK IMPROVEMENT** | 12 |
| 4 | **DREGEABILITY OF SLOPES AS PER DESIGN** | 13 |
| 5 | **ATTACHMENTS** | 14 |

ATTACHMENTS

| | |
|---|---|
| Att. 1 | **VP's travel schedule** |
| Att. 2 | **VP's questionnaire** |

## 0 SUMMARY

Dr.-Ing. Volker Patzold (VP) of ING.-BÜRO Dr.-Ing. V. PATZOLD, Holm-Seppensen/Germany, has been appointed by NHBDC to visit the site of New Naga Hammadi Barrage and Hydropower project during the period of January 21st til January 25th, 2003, and to inspect the dredging works for the diversion canal of said project and to give comments, if any, on

- The working methods of the dredging contractor in the diversion canal
- Identification of improvements of these working methods
- Assessment wether stable slopes as designed can be dredged

The contractor's working method is not adapted to prevailing soil conditions. This refers to

- The cut design
- The performance of design lines
- The operation of the reclamation area
- The productivity with impact on construction schedule

Improvements must focus on

- Definition of work still to be executed (volumes) from bulk and slope cut
- Revision of work method of slope dredging and adaption of equipment
- Work control
- Employment of more dredging equipment

Slopes can be dredged as designed, however, fine tuned dredging is necessary.

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# 1 INTRODUCTION

Dr.-Ing. Volker Patzold of Messrs. ING.-BÜRO DR.-ING. V. PATZOLD (VP); Holm-Seppensen/Germany, has been appointed by NHBDC to visit the site of New Naga Hammadi Barrage and Hydropower project during the period of January 21st til January 25th, 2003, and to inspect the dredging works for the diversion canal of said project and to give comments, if any, on

- The working methods of the dredging contractor in the diversion canal
- Identification of improvements of these working methods
- Assessment wether stable slopes as designed can be dredged

VP has received informations in

- Germany by Messrs. Lahmeyer International/Dipl.-Ing. W. Guth
- Egypt by NHBDC PM/Dr.-Ing. Kohli
- On site by
  - the NHBDC/site management/Mr. Hein, Mr. Erhardt,
  - the executing JV VINCI, BILFINGER BERGER and ORASCOM (the contractor), met on Jan 23rd, 2003, as well as
  - Messrs. ARAB CONTRACTORS, subcontracted by the contractor to execute among others the dredging works under discussion, met on Jan 22nd and 23rd, 2003.

# 2 WORKING METHOD

## 2.1 Dredging equipment

The contractor operates two cutter suction dredgers (CSD), make Ellicott/USA with following main data:

| CSD | El Safa | El Marwa |
|---|---|---|
| Series | 3000 Super Dragon | 1870 DRAGON |
| Dimensions (m) | 46,0*9,2*2,45 | 25*8,23*1,83 |
| Draught (m) | 1.7 | 1.22 |
| total installed power (kW) | 2880 | 1290 |
| dredge pump capacity (kW) | 1650 | 940 |
| Cutter capacity (kW) | 350 | 185 |
| Pipe diam (mm) | 600 | 500 |
| year of construction | 1980 | 1999 |

**Tab 1: Main data of Arab Contractor's CSDs operating in diversion canal**

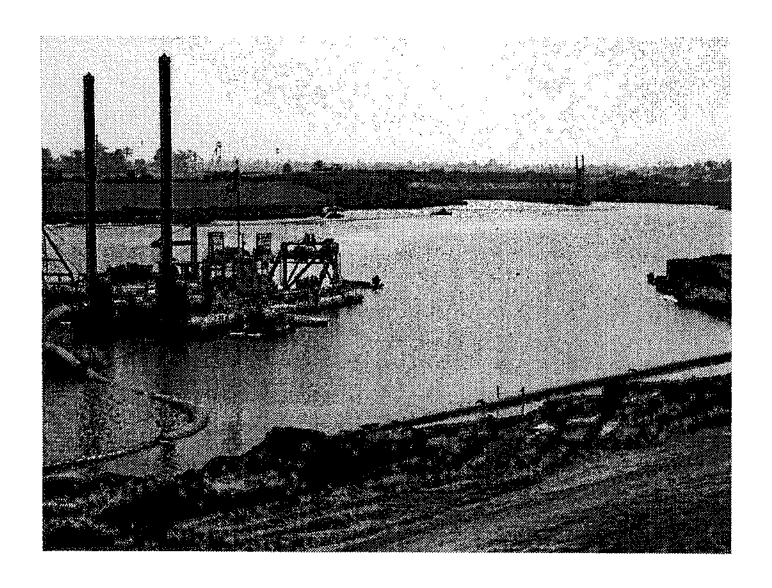


**fig. 1: CSDs El Marwa (left) and El Safa (right) dredging the diversion canal**

The CSDs seem to be in acceptable condition. The auxiliary floating equipment, too, seems to be in reasonable condition. Additional connectable floating pontoons have been mobilised to the site on Jan 23rd, 2003. Apparently they are old. The hull is covered by molluscs. NHBDC is advised to collect the proof of stability from the contractor under aspect of safety reasons.

The contractor explained that CSD El Marwa was scheduled to execute the slope dredging, and CSD El Safa the so called bulk dredging, that is dredging material from the canal's C/L - area (within an area of 10m off the the slope bottom lines).

**Remarks:** The CSD's design is old fashioned and due to the spud system used (s. fig. 1: spud stepping instead of advancing CSD via spud carriage with both spuds in CSD's C/L) they are less suitable, for slope dredging may be even unsuitable considering prevailing work conditions (for example regarding cut design, suction pipe mouth, pump force) .

## 2.2 Slope dredging

On occasion of the site visit on Jan 22nd, 2003, CSD El Marwa was working in the slope area dredging between elevation 59 and 57. The working spud was located 64 m (23.1.2003: 58 m) off the upper slope (elevation 67) resulting in a cut width of some 34m. The size of spud step executed was indicated by the contractor with 1,5m. When VP pointed out that this could be too much, the contractor explained a minimum step width of 0,9 m should be possible.

The dredge was swinging with a velocity of abt. 20 m/min. The contractor on January, 23rd, 2003, mentioned a speed of up to 29 m/min. This high speed apparently has been selected by the contractor to avoid longer lasting impact from the dredge pump on the slope. The CSD's cutter head diameter was said to be 1 m. VP pointed out that the cut height did not depend on the cutter's diameter.

El Marwa's inspected dredging works by the contractor were considered as trial dredging to compete with the prevailing soil conditions and the experienced slope failures. They aimed to find a proper dredging method to produce the slope with the designed inclination. According the contractor the method under trial differs from the initial work plan and foresees a 1st cut of 2,5 height due to the CSD's possibilities under prevailing low water levels. If less the CSD's pump would suck air. Once having finished the 1st cut, further slope shall be dredged in cuts of 1 m height instead of 2 m on 1st attempt.

Contractor will consider a pre-cut excavated by backhoe, to reduce the height of the 1st cut.

**Remarks:** The contractor has a wrong understanding of final levels to be dredged (s. annex 11, step 2 of his method statement, dated November 14th, 2002). This sketch clearly indicates contractor's intention not to dredge the designed depths. Some profiles from interim surveys of the works so far executed confirm this impression.

The contractor should be aware of the fact that by performing his revised slope dredging the final slope line would be located beyond the design line in direction to the canal's C/L.

Dredging the required slope includes overdredging. This overdredging, however, would create a slope failure of the embankment so far constructed, as the already finished area above elevation 59 will be undercut.

Once the slope is dredged in accordance with the method under trial right now, there remains still quite a volume to be dredged during a final dredging (due to the fact that the contractor works only the elevation 57 line). The contractor shall explain also the method and sequence of final dredging including the detailed stepping forward procedure.

In VP's opinion the contractor should minimize the specific volumes of the cut to be dredged. The contractor shall consider the consequence on productivity within his revised method statement.

In his working method the contractor has not identified consequences on the tolerance situation considering the dredge method of the CSDs (stepping). The contractor shall describe this situation in his revised method statement.

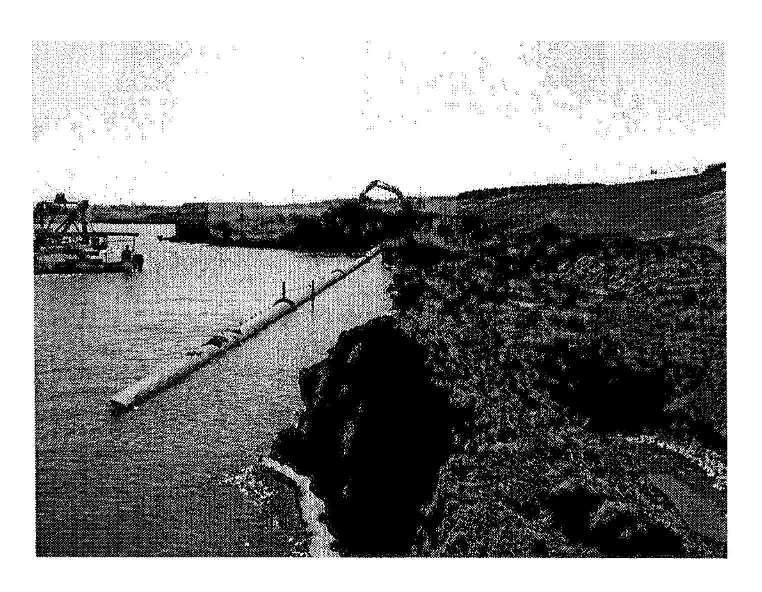


**fig. 2: method and result of slope trial dredging**
**(demonstrating the volumes for final dredging of slope prior to embankment protection works)**

**Conclusion:** The contractor shall revise his method statement especially regarding slope dredging, explain and demonstrate the different steps in detail and shall comment the consequences regarding the volumes to be dredged as well as impacts on time schedule and shall inform NHBDC without any further delay.

## 2.3 Discharge pipe and Reclamation area

The self-floating pipeline partially is of flexible type rubber hoses. The submerged pipe's construction is of rubber hose pipe, too, however without buoyancy layer. The submerged pipe should be controlled from time to time by echo sounding to register leakages, if any.

During the site visit of the reclamation area on January 23rd, 2003, 1 discharge pipe outlet was operating, however, the pipe was under water and the CSD was pumping against a sand wall under water (acting like a mineral fountain).The contractor explained that they could not suffer larger geodetic height unless reducing the pump capacity. This argument was refused by VP as not significant. Furthermore during discussions on Jan 23rd, 2003, it was revealed that the pump's impeller used was of 3 blade type. With 4 or 5 blades higher pressure easily could be reached. Arab Contractors have been advised to investigate this matter in more detail.

**Remarks:** A rough estimate made on Jan 24th, 2003 jointly Arab Contractors result in a required pressure of < 4,5 bar whereas > 4,8 bar are available on CSD El Safa.

So far no area reclaimed or levelled up to designed elevation could be recognized.

The kind of operation of the reclamation area is rather peculiar and definitely not in coincidence with the state of the art of dredging and reclamation works. Consequently to the work method the time schedule might be affected.

**Conclusion:** The contractor shall organize the reclamation area in accordance with the state of the art and contract conditions.

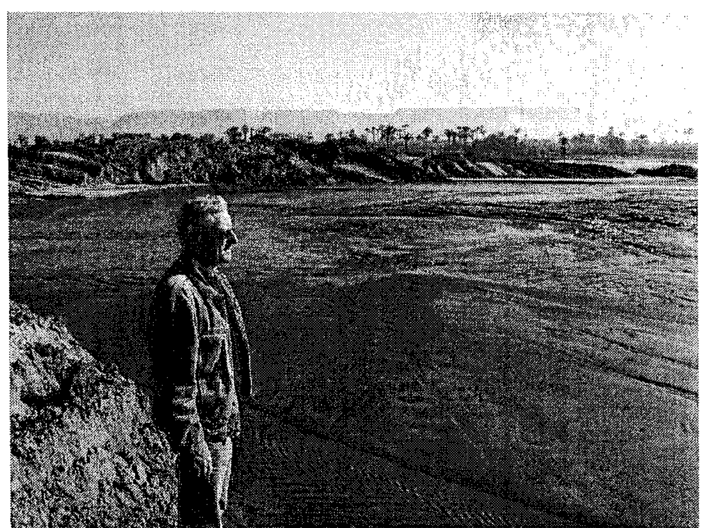


**fig. 3: irregular reclamation with submerged discharge pipe outlet**

## 2.4 Positioning and work control

The hydrographic work is executed by use of a Ratheon echo sounder, type 719 D, using 200 kHz single frequency. The contractor assured to use the same equipment during pre- and

post-dredging survey. NHBDC is advised to control this matter when present during interim and final survey. The hydrographic positioning is done by use of a RTK - system, which is acceptable.



**fig. 4: positioning of cutter at cut between elevation 59 and 57**
**(floating line marks border line at elevation 57, small beam indicates cutterhead position)**

The CSDs are daily positioned by use of a tachymeter, make Leica. For El Marwa's slope dredging the contractor since January 22nd, 2003, has developed a special positioning system by placing parallel to the slope a fixed floating pipe indicating for example the cut's bottom elevation 57. The dredge master now seems to have a more reliable indication of up to where to dredge. Continuous prove of the work executed, however, is not given.

Based on the experience so far made with a result of insufficient dredging work within the given limits NHBDC should decide in due time up to which extent they could accept contractor's work. This requires immediate presentation of the detailed planning und work method by the contractor, as requested during the meeting of Jan 23rd, 2003.

**Remarks:** For surveying a high frequency of 200 kHz is used. By this method already levels are indicated with material in suspension, whereas the solid ground could be in greater depth. VP recommends to NHBDC to make controls by comparison of depths sounded by echo sounder and lead as a first information. In case of differences a lower frequency ( for example15 kHz or 30 kHz) should used for control works. By this effort re-dredging eventually could be avoided or at least reduced.

The CSDs positioning system is antique and completely unsatisfying. VP recommended and explained to the contractor a modern method of dredge control, allowing on-line control of the works on board as well as in office. Under prevailing conditions NHBDC in VP's opinion definitely needs a even on-line documentation of the dredging works under execution. Otherwise a prove of quality is not possible.

**Conclusion:** More control work is necessary when dredging the slope area. The contractor immediately should indicate his improved positioning and work control system to NHBDC.

The contractor shall indicate in detail the way of embankment construction in accordance with performance of designed slopes.

## 2.5 Construction Schedule

According contractor's information on Jan 23rd, 2003, some 1,55 Mio m³ have to be dredged considering excessive volumes from dry excavation. Of this volume some 0,35 Mio m³ (22,5%) have to be dredged from slope area and some 1.2 Mio m³ from the bulk cut area.

**Remark:** The volumes mentioned by the contractor do not include any volume resulting from technically necessary overdredging. Those volumes still must be evaluated and indicated by the contractor. NHBDC is advised to check the volumes elaborated by the contractor.

In VP's opinion the contractor should estimate an average 30 cm overdredging in the canal's bottom area and 150 cm in the slope area. These estimated averaged tolerance volumes result in a total of abt. 130.000 m³ to be added to the volume still to be dredged (bottom line area: 1.200m*125m*0,3m=45.000m³; slope area: (1.600m + 600m)*25m*1,5m=82.500m³), incase design profile will be executed. In case of execution of a revised slope profile designed in accordance with the results of trial dredging only approx. 45.000 m³ have to be removed additionally.

According contractor's construction schedule the total volume to be dredged should be executed during the period September 1st 2002 until July 31st, 2003 (early finish = late finish). This means that diversion canal's dredging had to be completed within 11 months in accordance with original planning. During January 23rd's meeting the contractor explained that based on preliminary calculations the finishing date of dredging works could be postponed until September 30th, 2003, however, the milestone for operative start of river diversion (Oct 31st, 2003) would be kept. The latter revision, however, has not yet been approved by NHBDC.

During Jan 23rd, 2003, meeting the contractor presented a progress curve indicating a productivity ahead of schedule. VP doubts this information very much, as no work of the CSDs has been accepted yet nor has been executed any final dredging. The contractor after discussion agreed that the scheduled progress must be corrected and adapted especially because of the difficulties resulting from slope dredging.

On basis of the contractor's updated hydrographic survey of December 19th, 2002, some 0,5 Mio $m^3$ have been estimated since actual start of dredging works on October 27th, 2002, until VP's visit. The further analysis results in a performed weekly brutto production of abt. 41.670 $m^3$/W (assuming 12 weeks dredging period). The planned production, however, should have been abt. 669.300$m^3$ paid volume and finished work, corresponding to 55.775 $m^3$/W. (1.550.000 $m^3$/11 mth = 140.900$m^3$/mth*4,75 mth = 647.730 $m^3$ + tolerance dredging volumes).

Contractor explained on Jan 23rd, 2003, that productivity of CSD El Safa suffered a stop of abt. 21 days due to reduced productivity of dry excavation works. Insufficient area had been stripped from overburden.

**Remarks:** The bulk dredging is estimated by the contractor according his verbal information with 5.000 $m^3$/d. The remaining volume to be dredged is abt. 700.000 $m^3$ corresponding to abt. 5 months work period. Reasonable time for final dredging of the bottom of 1 month should be added. Based on the experience so far made the bulk dredging will take another 6 months from end of January 2003.

In slope dredging the contractor expects according his preliminary information a daily production 1.000 til 2.000 $m^3$. Based on an average of 1.500 $m^3$/d the remaining dredging period will be abt. 5 months assuming a remaining volume of 350.000 $m^3$ plus a provision for final dredging of estimated 1,5 months, resulting in a period of 9,5 months from now on. This preliminary estimate even exceeds the extension of construction up to September 30th, 2003. NHBDC must expect further delay due to reduced productivity in case of decreased cut conditions (cut height, stepping).

Tolerance dredging must be considered, too.

Unfortunately the dredged volumes invoiced up to now mainly result from the diversion canal's bulk cut. No part so far dredged neither from bulk nor slope cut can be considered as already finished work.

**Conclusion:** The contractor in VP's opinion apparently is behind schedule. Much more delay must be expected due to more time consuming dredging in the slope area as well as final dredging of main cut. The contractor shall present a revised and detailed time schedule to NHBDC.

## 3 PROPOSALS ON WORK IMPROVEMENT

NHBDC should control contractor's work more intensively. Contractor's reporting system, if any, should be analysed more intensively by use of qualified experience. The more this becomes necessary as apparently the contractor has not sufficient dredging expertise available on site to control his subcontractor's (Arab Contractor, Cairo) work.

In general the contractor must install suitable work control systems, identifying the works to be executed directly to the dredge master and on-line to the survey department and management. Such work control could be realised by the system MARPO-DGPS, produced by Messrs. ARGE VPC & SPE, Holm-Seppensen/Germany (a brochure describing the system in detail was handed over to NHBDC/Dr. Kohli and Arab Contractor/Mr. Tarek).

Regarding the slope dredging, in VP's opinion the specific cut volumes must be reduced by adaptation of cut dimensions. Furthermore CSD's El Marwa suction capacity eventually should be reduced.

To increase productivity the slope dredging should be executed by at least CSDs, may be of smaller size than CSD El Marwa. Those CSDs could pump the material dredged in the bulk cut area, from where the material is removed by El Marwa/ElSafa.

In case slopes can not dredged by CSD, that means hydro-mechanically, the winning method must be changed to mechanical cutting by use of a clam shell or a bucket, depositing the excavated material in front of the CSDs. The decision on the dredging method to be performed must be taken without any further delay.

The contractor should definitely consider the contract clauses with special regard to the lines and levels to be delivered.

A final proposal on improvements can not be indicated at this moment as the contractor at first has to answer the questions as listed in the questionnaire prepared by VP (s. att. 2).

## 4 DREGEABILITY OF SLOPES AS PER DESIGN

During initial slope dredging two slope failures were experienced. NHBDC by studies have proven, that the slopes in consideration of the relevant geo-technical parameters are stable.



**fig. 5: slope failure #1 at northern entrance of canal**

NHBDC's calculations result in safety factors of h=1,41 til h= 1,81, depending on the load combination assumed (s. Tender design report Vol.1 of September 2000). The safety factor of slope stabilty of h = 1,4, to be applied for normal load cases must be exceeded, what happened in all load combination cases analysed.

The slope in VP's opinion and experience in principle can be dredged with an inclination 1:3 = H:L in case an adapted dredge programme is executed. A detailed soil investigation is considered necessary to get more information on th4e layer's location and variation. Once the final work method is elaborated, it might be revealed that the contractor does not stick to the requirements during execution. NHBDC must be able to control contractor's work.

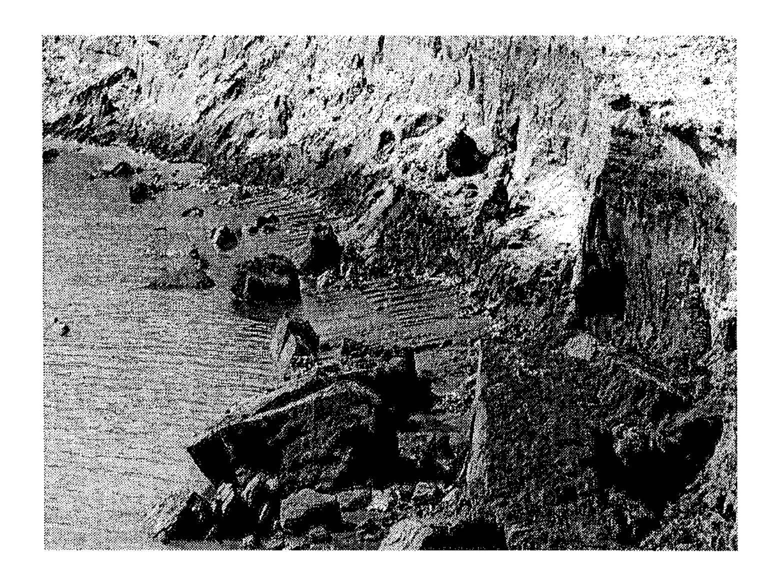


**fig 6: soil to dredged (at slope failure #2)**

The contractor explained during the meeting on Jan 23rd, 2003, his opinion that all efforts could fail, at least in some areas. The question was raised, what the solution could be in such circumstance. VP explained, that the contractor already should have defined such areas by suitable soil investigation methods, for example by sufficiently calibrated geophysical resistivity surveys, to adapt his dredging activity to those sensible areas. Results of such investigations should be discussed. Further details should be elaborated as soon as the questionnaire has been answered by the contractor up to the complete satisfaction of NHBDC.

## 5 ATTACHMENTS

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Holm-Seppensen, January 26th, 2003

Dr.-Ing. Volker Patzold
Beratender Ingenieur VBI

**Attachment 1**

VP's travel schedule

## VP's TRAVEL PROGRAMME

16.01.2003 preparatory meeting with NHBDC in Frankfurt/Germany
preliminary info on project

21.01.2003 travel to Cairo/Egypt
meeting with NHBDC's project manager Dr.-Ing. Kohli, info on project

22.01.2003 site
meeting with NHBDC's site management, receipt of documents related to dredging of diversion canal, introduction of VP to NBHDC's client,
site visit, inspection of dredgers, discussion with contractor, preparation of questionnaire for contractor

23.01.2003 site
site inspection, inspection of reclamation area, discussion of contractual matters with NHBDC's contract engineer (a.o.: consequences of acceptance of method statement by NHBDC)
discussion of questionnaire with contractor

24.01.2003 site
preparation of memo, discussion of memo

25.01.2003 return to Germany
discussion of memo in Cairo with PM, CRE and contractor

26.1.2003 Holm-Seppensen
preparation of final report

**Attachment 2**

VP's questionnaire
presented to contractor on January 22nd, 2003, by NHBDC

NAGA HAMMADI BARRAGE DEVELOPMENT CONSULTANTS

Questionnaire

Subj.: Diversion Canal Dredging Works

**1 Dredging volume**

1.1 total volume to be dredged after execution of dry excavation
- 1.1.1 volume from diversion canal cut (bulk dredging)
- 1.1.2 volume from slope area
- 1.1.3 volume from tolerance dredging
- 1.1.4 planned output for canal and slope dredging

**2 Construction schedule**

2.1 period of diversion canal cut dredging
2.2 period of slope dredging
2.3 actual status of dredging works (canal and slope dredging)

**3 Dredging Technique**

3.1 define assumed overdredging thickness in canal and slope area
3.2 provide detailed description of dredging of slope area including 10 m safety area
- 3.2.1 description of foreseen cut height considering different soil conditions and type/size of cutterhead
- 3.2.2 indication of swing velocity, spud-step width, diameter and rounds of cutter head, pump capacity m3/h, density of soil-water-mixture
- 3.2.3 estimate of volume to be overdredged considering the spud-step technique of CSDs used
- 3.2.4 estimate on daily net output (paid production) in slope dredging in accordance with proposed method
- 3.2.5 comment expected bottom line as described in the method statement (annex 11, step 2)
- 3.2.6 indicate in detail the sweeping method including the equipment description and considering the unevenness of bottom line after dredge performance
  - 3.2.6.1 in diversion canal cut
  - 3.2.6.2 in slope area

**4 Dredge Control**

4.1 description of dredge control system to be operated
4.2 description of work control system to be performed
4.3 description of hydrographic survey system indicating

4.3.1 type and frequency of echosounder used

4.3.2 type of positioning system used incl. indication of accuracy (in case of DGPS indicate code used)

4.4 indicate survey programme considered necessary for dredge control

Naga Hammadi, 22 January 2003

ARAB REPUBLIC OF EGYPT
MINISTRY OF WATER RESOURCES AND IRRIGATION
MINISTRY OF ELECTRICITY AND ENERGY
HYDRO-POWER PLANTS EXECUTIVE AUTHORITY

**NEW NAGA HAMMADI BARRAGE AND HYDROPOWER PLANT PROJECT**

# Study of Slope Failures During Excavation of the Diversion Canal: A Case Study

By

Geotechnical Consultants

**Dr. Abdel Fattah Abouleid**
Ahmed Amin St., Dokki, Giza.
Tel 335 3282

**Dr. Mostafa Mossaad**
EGEC, Abuhazem St., Haram, Giza.
Tel. 582 9685

Submitted to

RGBS-Site Resident Engineer

Project No.: 2382/34-2003 (0203145)

January 2004

## Table of Contents

1. INTRODUCTION 2

2. APPROACH TO ANALYTICAL INVESTIGATION 3

3. DETAILS OF ANALYTICAL INVESTIGATION 4

3.1 Scope 4

3.2 Task 1: Analytical Investigation on Failures in the Dredged Slope 4

3.2.1 Concept 4

3.2.2 Analysis and results 5

3.3 Task 2: Analytical Investigation of Cracks on the Slopes formed by Excavator 7

3.3.1 Concept 7

3.3.2 Analysis and results 7

4. CONCLUSIONS 9

## Appendices

Appendix 1: Results of static safety analysis.

Appendix 2: Results of the Analysis of the dredged slope.

Appendix 3: Results of the Analysis of the excavated slope.

Appendix 4: Surveyed failure of the dredged slope.

# 1. INTRODUCTION

This report has been prepared upon the request of the RGBS – site Resident Engineer, for the New Naga Hammady Barrage Project. During the excavation for the diversion canal, several slope failures took place, and some change in the initial design slope geometry had to be made after a number of trials with excavation/dredging equipment. The objective of this report is, therefore, to present the results of an analytical investigation of the slope failures problem as to its probable causes in relation to the existing soil conditions and/or the excavation/dredging equipment used.

A copy of some documents pertaining to the subject problem was made available to us, these include:

- A CD containing some photos of the slope failures.
- Design criteria, and calculations for earth slope stability.
- Borehole logs from a number of geotechnical investigations carried out in the diversion canal area.
- Method statement submitted by the dredging contractor.
- Correspondences between the Engineer and the Contractor pertaining to the slope failure problem.
- Surveying of the slope failure sites showing failure profiles.
- A report based on a site visit made by a dredging expert.
- Alternative slope geometries proposed by the contractor
- Observations made during an excavation trial by means of a long boom backhoe.

In addition to the above documents, which have been examined carefully, a site visit was made on October 18, 2003. During the visit, a detailed discussion with the client's engineers took place, and a number of samples from excavated material was requested during that visit, and were received and examined two weeks later.

The above items/activities formed the basis of the present investigation, which aims at exploring the potential impact of site/work conditions on the slope failures problem.

## 2. APPROACH TO ANALYTICAL INVESTIGATION

In the present study, an attempt was made to make use of the widely known geotechnical finite element software, PLAXIS, to simulate the site/work conditions and see if the results were comparable to those realized on site, as depicted from the reported surveying measurements and photos.

It is a well-known fact, however, that modeling must necessarily involve some simplifications and assumptions. The present work is no exception, as the problem under consideration is indeed far less common than many modeling problems handled by similar geotechnical analysis softwares.

The present investigation is divided into two modeling tasks, which are aimed at:

1) Analyzing the process of suction dredging of sand layers, and its potential relation to the reported slope failures
2) Examining the stability of the side slope, in light of the observations made during the site visit and the cracks that were observed in the excavation trial, made by the long boom backhoe excavator.

In both modeling attempts, the side slope profile used in the design calculations made by the Designer was adopted. An attempt was made, however, to vary the design soil parameters and examine the sensitivity of the results to these parameters. The modeling details, results and interpretation are presented in the following sections.

# 3. DETAILS OF ANALYTICAL INVESTIGATION

## 3.1 Scope

As previously indicated, two problems relevant to the canal construction works are analyzed in this report. The first is the one of the slope failures that occurred in the parts of the canal, in which suction cutter head dredgers were employed to form the canal slopes. The second is the problem of the cracks that appeared in the slopes during the trial test, in which mechanical excavators were used instead of suction dredgers to form the required slopes.

## 3.2 Task 1: Analytical Investigation of Failures in the Dredged Slope

### 3.2.1 Concept

According to the available documents, the diversion canal slope is typically 3H:1V; the slope crest elevation is around 67.0 m whereas the bed elevation is around 52.00 m. The water elevation is approximately at elevation 58.0 m. The top 8.0 to 9.0 m of the embankment consist mainly of Holocene silt to fine sandy silt. The underlying layer is a thick fine to medium sand deposit that extends far below the bed level. A typical section of Pleistocene the analyzed slope is depicted by Figure 1 below.

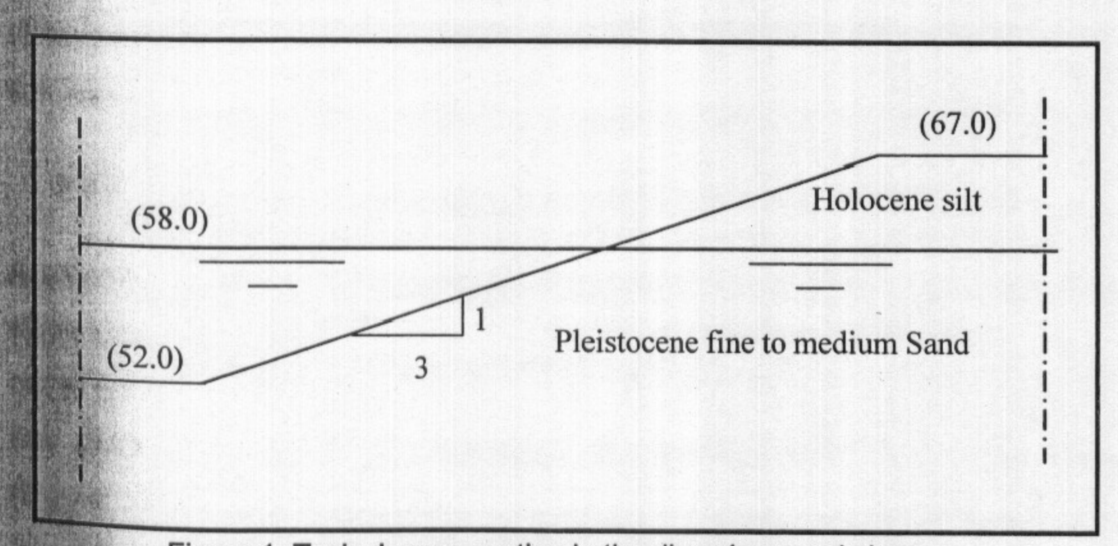



Figure 1: Typical cross section in the diversion canal slope

General experiences as well as simple calculations suggest that a 3H: 1V slope in loose to medium sands is a safe slope with adequate margin of safety under hydrostatic conditions. Assuming no lateral seepage through the investigated slope, two scenarios were found possible, in principal, to cause the reported failures. The first is the undercutting in the bottom sandy part of the slope, presumably due to over-dredging. The second is the loss of soil strength due to the effect of the suction pressure generated by the dredgers. Whereas the first scenario is less likely to happen, if proper operation and monitoring of dredging process is maintained, the second seems inevitable, as the hydraulic effects caused by the dredger's suction pressure cannot be avoided. Accordingly, our analysis has focused on the potential impact of suction on slope stability.

Suction dredging is, in essence, employed to remove soil particles, and draw them in the form of a sediment load into long disposal pipes to dump them some distance away from their original position. Therefore, the effect of a suction applied at the surface of a submerged sandy deposit may be viewed as a process of “accelerated piping failure” that has its peak effect at the surface, and extends into the submerged soil mass for some distance proportional, essentially, to the magnitude of suction applied. With this concept in mind, simplified calculations for a case of one-dimensional flow would readily show that a suction of 0.5 to 0.6 bar could result in the diminish of effective stresses, and consequently the loss of strength, for a soil column of about 7.0 m depth or more.

### 3.2.2 Analysis and results

A preliminary slope stability analysis under the static condition, using the same design slope and without considering the effect of suction was carried out to verify the finite element results. The results of this verification analysis are given in Appendix 1. The resulting safety factors, 1.40 for c=0 (Figure 1.1) and 1.71 for c=20 kN/m$^2$ (Figure 1.3), were found to agree with the corresponding value of 1.75 reported in the Design Calculations. The resulting failure surfaces

A finite element model was then constructed to simulate the suction-triggered failures, and to capture the potential failure surface according to this stipulation. In modeling the effect of suction, an assumed suction "bulb" of influence within the soil mass was employed. In dredging from bottom to top in the sandy layer, it is believed that the critical case occurs when the dredger operates at the upper part of the layer after having dredged the lower part. In this case, the upper part of the sand layer turns into a state of liquefaction, while the lower part is already significantly loosened up. The soil parameters adopted in the analysis are shown in Table 1.

Table 1: Modeling parameters for the site soils

| Soil Stratum | Thickness (m) | C (KN/m$^2$) | $\phi^o$ |
|---|---|---|---|
| **Holocene Silt** | 9.0 | 0 to 20 | 25° |
| **Silty Sand** | Extended | 0 | 32° |

The results of this analysis are presented in Appendix 2. The slope model and the location of the "suction bulb" are shown in Figure 2.2. The stages of analysis and the soil parameters are given in the tables of Figures 2.1 and 2.3. respectively. Figures 2.4 through 2.6 show the deformed mesh, the incremental displacement contours and the location of shear and tensile failure spots, respectively for the case with c= 20 KN/m$^2$. The results of the more critical case of c=0 are presented in Figures 2.7 through 2.9. In both cases, however, the shape of failure zones are found to be reasonably comparable to those determined from post-failure surveying measurements made by the contractor and reproduced in Appendix (4) for reference.

It has to be noted that the possible occurrence of seepage lines at the failed spots may also have contributed to the reported failures. Contrariwise, some other stabilizing factors such as the absence of seepage forces, the deeper extent of the top cohesive layer and, possibly, the larger cohesion values for either the top or the bottom strata than the estimated ones may have prevented the failure of other non-failing spots. It is also believed that the speed of dredging may have played a role in the occurrence of failures.

## 3.3 Task (2): Analytical Investigation of Cracks in the Slopes formed by Excavator

### 3.3.1 Concept

As previously mentioned, both general experience and simple calculations suggest that a 3H: 1V slope in loose to medium sands be adequately safe under static loading conditions. In the field test segment, however, and during forming the canal slope to the required inclination using a mechanical excavator, longitudinal cracks appeared around the elevation 63.0 to 64.0 and the test was discontinued by the Engineer, apparently to preclude what seemed to be a potential failure. The formation of relatively wide cracks during excavation seemed rather puzzling as the static factor of safety of the finished slope is more than adequate (approximately 1.71). It's also reasonable to assume that the test was closely monitored by the Engineer, as to prevent accidental excessive over excavation that may lead to slope failure due to under cutting for the upper cohesive layer.

On the other hand, some tensile hair cracks in the upper cohesive layer must be expected to develop due to the stress relief resulting from the mere process of excavation. Nevertheless, we find it difficult to explain the occurrence of the reported wide cracks by the previous argument. Accordingly, an attempt was made to explore the possibility of other destabilizing factor(s) that may have reduced the relatively large value of static factor of safety, and contributed, significantly, to the near-failure condition encountered during the test with excavator. In our discussion with the Client, during our site visit, it was confirmed that clearly visible seepage was observed through the part of the slope above water during excavation. It was further explained that the area atop the subject slope was an agricultural land used for growing water-demanding crops such as rice. Based on this finding, the following analysis was carried out.

### 3.3.2 Analysis and results

Two finite element models were constructed to simulate the excavated slope. In the first model, initial tension cracks caused by the shrinkage of

the drying soil surface due to summer heat were modeled at an elevation similar to that reported in the field. In this case, the slope is further acted upon by an assumed seepage line. In the second model, only the seepage line was considered to act on the slope.

The results of these analyses, with c=20 kN/m$^2$, for the cases with and without cracks are presented in Appendix 3. It should be noted that in the case "without crack", the crack effect was nullified numerically, although its boundaries still exists, as shown in the figures pertaining to that case. The slope models before the application of the seepage line for the cases with and without initial cracks are shown in Figures 3.2 and 3.9. For the two models, the stages of analysis are given in the tables of Figures 3.1 and 3.8, while the material parameters are presented in the tables of Figures 3.3 and 3.10. Figures 3.4 through 3.7 and 3.11 through 3.14 show the deformed mesh, the incremental displacement contours and the location of shear and tensile failure spots for the two analyses, respectively.

It was found, however, that in both cases, with and without initial cracks, tensile failure points appeared at the surface of the slope (Figures 3.6, 3.7, 3.13 and 3.14). Also, many points critical in shear occurred within the potential failure zone (Figures 3.6, 3.7, 3.13 and 3.14). The difference between the two analyzed cases was the appearance of tensile failure points in the initial crack at elevation 63.50 in the first model (Figure 3.7). These results indicate that irrespective of the existence of initial cracks, subsequent tensile, shear and tensile-shear cracks could develop in a 3H: 1V slope under the effect of the assumed seepage condition.

## 4. CONCLUSIONS

Two slope failure problems encountered during the excavation of the Diversion Canal are analytically investigated in this report. In the first problem, failures occurred in some parts of the canal, in which suction cutterhead dredgers were employed to form the canal slopes. In the second, a near-failure condition is believed to have developed, as manifested by wide longitudinal cracks in the slopes of the test segment, in which mechanical excavators were used in shaping the slopes, around the elevation range 63.0-m through 64.0.

Mathematical models were constructed using the well-tested and widely known finite element code, PLAXIS, to simulate the reported symptoms of the subject problems.

A preliminary verification analysis, was first carried out to check the existence of common grounds between the present analysis and the stability calculations of the Tender Documents. The analysis showed that the resulting factor of safety is in agreement with the design safety factor.

As for the first problem, two scenarios were seen possible to cause the reported failures. The first is the possible undercutting in the bottom sandy part of the slope, presumably due to over-dredging. The second is the loss of effective stress, and consequently soil strength, under the effect of the suction pressure from the employed dredgers. We do believe that there are no significant reasons to exclude the probability that some over dredging may have taken place. Nevertheless, it is hard to believe that limited undercutting would lead to failure zones of the extent shown by the surveying measurements. In the second scenario, however, our analysis indicates that the loss of strength could extend to a depth of 7.0 m or more. This can, according to the performed analyses, cause slope failures comparable to the actual ones. This implies, in our opinion, that irrespective of whether or not over-cutting has occurred, failure might have been more strongly triggered by loss of strength in the submerged sandy part of the slope.

As for the second problem, our results indicate that irrespective of the existence of initial cracks, subsequent tensile, shear or tensile-shear cracks could happen in a 3H:1V slope if a significant lateral seepage occurred. A final assessment of the case can be made if the real profile of the seepage line is determined.

In summary, the results of the above analysis suggest the following:

a) The loss of strength in the submerged part of the sandy layer due to the suction pressure from the used dredger is the principal cause of the reported failures. Not having failure in all segments excavated by suction dredging could be attributed to a number of stabilizing factors including the absence.

b) The effect of the seepage forces, not the mechanical excavator, may have been the cause of the wide crack in the test segment, due to the reduction in the factor of safety.

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